



## D5.4 Guidelines for interaction between seabed and support structure

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## D5.4 Guidelines for interaction between seabed and support structure

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## 1 Executive Summary

Geotechnical aspects of MUPs have been researched as part of the MERMAID project, with particular focus on the stability of foundation soils and their necessary scour protection. Details of this work are published in a number of publications as provided in the reference list. Practitioners will be particularly drawn to the book “Liquefaction around Marine Structures” which covers many of the aforementioned subjects researched within MERMAID regarding liquefaction issues of platform designs (Sumer, 2014). The principal guidelines emerging from the complete body of work are:

- Sandy soils have a high liquefaction potential, the effect of which must be accounted for in foundation design.
- The presence of small quantities of fines within a sandy soil can act to *increase* the potential for wave-induced liquefaction. For the soils studied, this occurred for fine fractions up to 30% of total mass. Sandy soils with larger quantities of fines, or sandy clays, were less susceptible as the clay behaviour dominates above this fraction.
- Liquefaction risk under gravity base foundations should be screened properly in the light of in-situ and lab tests of the seabed soil. The preliminary screening methods in the literature (some of which cited here) can be used as a first assessment. If these assessments indicate any risks, detailed analyses and models should be applied for liquefaction assessment.
- When it comes to wave-induced liquefaction, standing waves would be seen in the vicinity of reflecting boundaries of platform foundations. Liquefaction under Standing waves, although qualitatively similar, show features different from that caused by progressive waves. The pore pressure accumulation and liquefaction starts at the nodal section, and progresses towards anti-nodal section due to a diffusion mechanism. The rate of liquefaction at the nodal section seems to be the same with the progressive wave case. Thus, mathematical models for progressive waves would also work for standing waves as far as the nodal section is concerned.
- Rock berms (protection cover) on pipelines and power transmission cables are essential to protect the structure from floating to the seabed surface due to liquefaction, and strongly suggested to be constructed. However, once the rock berm is constructed, care must be exercised on backfilling the excavated soil into the trench intentionally or due to sediment transport.
- Studies showed that a cover of stones constructed around the platform foundations for scour protection purpose would also serve for increasing the liquefaction resistance of surrounding soil.
- However, the stability of such stones against sinkage must also be assured, and studies are ongoing to provide guidelines for this too, which should be ready before the completion of the project (before M48).

## 2 Introduction

This report discusses the context of deliverable D5.4 – Guidelines for Interaction between seabed and support structure, with respect to the MERMAID project. The deliverable will discuss on the geotechnical challenges that the MUPs could face due to the effects of waves, currents, seismic loads, and/or a combination of them. A set of guidelines are provided which are a results of physical model experiments carried on during the MERMAID project. The most important mechanical processes occurring in typical foundation configurations for MUPs will be described.

### 2.1 Scope of Deliverable D5.4

The objective of Task 5.4 is to understand the main important geotechnical issues related to the technical feasibility of the MUP under MERMAID. A focus is given to the phenomena of liquefaction produced by waves and its effect on scour protections around foundations. A series of physical model experiments and numerical modelling has been conducted by researchers as part of the MERMAID project: B. Mutlu Sumer (DTU-MEK), Ozgur Kirca (ITU) using a wave flume facility, Andrew Brennan (UoD) using a dynamic centrifuge equipment, and Waldemar Świdziński & Andrzej Sawicki (IBWPAN) conducting 1-g physical modelling and a wave flume facility. A literature review and a summary of guidelines is given through the research conducted by the contributors, published or not.

The main objectives of this task are to support the design of the MUPs with emphasis in liquefaction analysis. Important problems of wave-induced liquefaction are covered, such as: residual liquefaction, momentary liquefaction, liquefaction in clay and sand mixtures, and liquefaction in standing waves. As well, scour protections are considered relevant and have been studied with experimental tests for wave-induced liquefaction effects over cover stones around foundations. In the present UoD is conducting centrifuge tests (A. Brennan & D. Escibano) and ITU (S. Kapdasli & O. Kirca) is conducting shaking table tests to study seismic induced liquefaction effects on cover stones. The tests also include the case of seismic-induced liquefaction on offshore foundations with no scour protection. Numerical model tests (D. Jeng, UoD) will then be calibrated/validated in order to design charts used as an output of the project.

A 2D and 3D numerical model has also been developed through the MERMAID project by Professor Dong-sheng Jeng (UoD). In the existing model, wave propagation around marine infrastructures can be simulated, as well as the evaluation of the seabed response (including pore pressure and effective stresses; liquefaction etc.) under dynamic loading. Both poro-elastic and poro-elastoplastic soil constitution have been included in the new version. In this project, the model will be further applied to more complicated configuration of marine infrastructures, for example, the Seed Mussel Capture Devices in Wadden Sea. At IBW PAN a compaction/liquefaction model has been developed, with three differential equations in the form of constitutive laws (compaction law, stress-strain relationship and relation between compaction and pore water pressure generation) with some material constants which have to be determined experimentally for a given soil.

### 3 Study on Seabed Characteristics

Physical and mechanical characteristics of seabed are essential for proper designing of foundations of platforms and other structures forming the marine infrastructure.

Typical foundation types (see D3.2 on shallow, deep and ultra-deep water foundation technologies, and section 3, below) will be piled or gravity based for the fixed structures, and anchored moorings for the floating structures. The loading on the foundation will be periodic and of many cycles. Therefore the seabed characteristics of interest are the responses of these soils to cyclic loading. Seabed characteristics at the sites showed a prevailing presence of sandy soils. In studying the cyclic behaviour of sandy soils, the key soil characteristics will be those related to liquefaction – the accumulation of excess pore pressures over a number of cycles, consequently reducing the effective stress to near-zero values.

Sawicki, Kazimierowicz-Frankowska & Zabuski (2014) summarized the materials in the Krieger's Flak and the Wadden Sea, and identify a methodology for assessing liquefaction potential in such soils. However, as the aim of the project is to evaluate not just the four specific cases but the capacity for more general deployment, it was deemed prudent to identify a soil capable of representing the behaviour of a range of sandy seabed's rather than simply the specific soils found at particular sites. For repeatability, and so that subsequent model tests may be carried out on the same soils, two uniformly graded sands and one type of silt were identified as a suitable base material. The sands were HST95, a fine, uniformly graded sand used previously in many studies as a representative soil for offshore conditions, and another similar fine sand (ITU Sand, Table 1) that is artificially produced/grinded and commercially available in Turkey (referred as ITU sand). Whilst the sands were characterized in some detail by Sawicki, Mierczyński & Sławińska (2014), the silt was used in the DTU flume tests by Sumer et al. (2012), Kirca et al. (2012, 2013, 2014) and Kirca (2013). Some key characteristics are provided in Table 1, below, and guidance presented within this report is largely based around the response of these soils.

*Table 1. Salient soil characteristics*

Soil Property	Sand Type		Silt Type DTU Silt
	ITU Sand	HST 95	
D <sub>10</sub> (mm)	0.09	0.10	0.023
D <sub>30</sub> (mm)	0.11	0.12	0.050
D <sub>60</sub> (mm)	0.14	0.14	0.073
C <sub>u</sub>	1.5	1.4	3.2
C <sub>z</sub>	1.0	1.0	1.5
G <sub>s</sub>	2.65	2.63	2.67
γ <sub>dmin</sub> (kN/m <sup>3</sup> )	12.44	14.59	11.91
γ <sub>dmax</sub> (kN/m <sup>3</sup> )	15.32	17.58	16.68
e <sub>max</sub>	1.092	0.769	1.20
e <sub>min</sub>	0.699	0.467	0.57
k (m/s) in loose		1.23x10 <sup>-4</sup>	1.50x10 <sup>-5</sup>
Shape	Rounded	Rounded	Rounded
φ <sub>crit</sub> (o)	34.4	32	35.2

Extrapolation of results to other projects should therefore only be carried out with the comparison between soil types, and engineering judgment is recommended in determining the degree to which another soil will replicate the behaviour observed when producing this data.

Further studies on the sensitivity of this soil to the presence of finer fractions, on the relationship between wave loading and long term stability and on the stability of coarser soils were also carried out for generality, and are summarized in section 4 and 5, below.

## 4 Geotechnical challenges of MUPs

Multi-Use Offshore Platforms (MUPs) carry out new geotechnical challenges in marine engineering as new functions of these structures will require different foundations that will be subjected to various loadings, including hydrodynamic and seismic loads, depending on their location (Sawicki et al., 2014c). The foundations for MUPs may involve a wide range of configurations from fixed steel or concrete platforms, to floating facilities. For large water depths it is necessary to study different anchoring systems depending on the different loading regimes imposed.

Pile or jacket foundations are commonly used for fixed offshore platforms. They are designed to resist the deadweight of the structure and environmental loads, such as cyclic loading which imposes tensile and compressive loads to the piles, as well as lateral loading (Randolph et al., 2005). Piles can also be used as anchors for floating foundations. Shallow foundations skirted gravity bases and bucket foundations which are installed under self-weight, as well as caissons that are penetrated in the seabed by suction. Similar to pile foundations, shallow foundations have to withstand important cyclic horizontal and vertical loads that can generate excess pore water pressures in the seabed. Floating facilities are connected to the seabed through a mooring system connected to an anchor. Anchors are also used to provide extra stability to fixed or flexible structures (Randolph et al., 2005).

Important aspects that have been considered particular attention for the MERMAID project are:

- Flow in porous structures (scour protection, flow under GBS, flow through mussel farms and cages)
- Soil liquefaction and platform interaction
- Risk of liquefaction and the sinking of cover stones and heavy structures due to liquefaction
- Seismic/hydrodynamic originated cyclic loads or vibrations
- Effect of clay content on processes
- Support to foundations and mooring systems

Under the MERMAID project liquefaction has been broadly investigated by Sumer et al. (2010, 2012) and Sumer (2014), Kirca et al. (2012, 2013, 2014), Kirca (2013), Jeng and Zhao (2014), Sawicki et al. (2014a, 2014b, 2014c, 2014d, 2014e, 2014f), and Sawicki & Sławińska (2014). Other processes that need attention are those involving flow-structure-seabed interactions, such as (Sumer, 2014b):

- Turbulent boundary layer over the seabed,
- Boundary layers over the surface(s) of the structure;
- Boundary layer separation and lee-wake processes from and around the structure;

- Sediment transport and scour/erosion processes;
- Various dynamic effects including wave-impact forces on structures;
- Various geotechnical-engineering processes such as liquefaction.

Geotechnical site investigation is an important aspect that has to be considered, as well as a laboratory testing program. This task involves high costs and therefore it is not always possible to perform a detailed investigation of the soil conditions. A standard site investigation will require an experienced geotechnical engineer who would select specific sites for in situ tests such as boreholes, CPTs, and obtaining soil samples for laboratory testing. With these tests it is then possible to evaluate the liquefaction potential of a site.

## 5 Liquefaction around MUPs

Liquefaction describes the failure of a saturated soil when high pore pressures are generated, resulting in large strains and continuous deformation. A liquefied soil will behave as a viscous liquid, and there the term liquefaction. Seed (1976) defined liquefaction as the condition where soil experiences continuous deformation at a constant low residual strength or with no resistance at all, due to build-up of pore water pressure. By the application of either static or cyclic loads pore water pressure can build-up in the soil. This phenomenon is observed in soil deposits subjected to undrained loading and with a loose density. The phenomenon of liquefaction will influence the behaviour of marine structures founded on the seabed, as it may lead to instability of structures, sinking of the liquefied seabed, floating of pipelines, massive slides, etc. (Sawicki, 2014).

Earthquake loads and strong storms are the major causes of liquefaction in marine deposits. The main differences between wave and earthquake loading are that wave loading has a longer period, lower frequency (0.1Hz), and the stress fluctuation is different (De Groot et al., 2006a). As well, in the case of a storm, the loading propagates from the seabed into the subsoil and has a relatively longer period compared with an earthquake. On the other hand, earthquake loading propagates from the ground and moving up to the mud line.

Evidence of liquefaction in offshore structures has been reported in the literature. Christian et al. (1974), and Herbich et al. (1984) have reported the phenomenon of floatation of pipelines due to storms, produced by liquefaction of the seabed. Miyamoto et al. (1989) reported the subsidence of offshore breakwaters at the Niagata Coast, Japan. Sawicki & Mierczynski (2006) and Sawicki (2014) provide an extensive literature review regarding the practical implications of liquefaction and the dynamics of seabed's and marine structures. Reports on earthquake induced liquefaction in marine structures has been well documented summarized by Sumer et al. (2007). In the 1999 Turkey Kocaeli Earthquake several cases of liquefaction were observed in marine structures such as: behind quay wall and sheet-piled structures, settlement, tilting, and even collapse of structures near the shoreline, slope failures, and lateral spreading.

Gravity based foundations, such as the type of caisson breakwater (Figure 1), may encounter liquefaction produced by standing waves (see section 4.2.1), and under the structure. Under the caisson, pore water pressure can build up by wave motion and by caisson motion (Kudella et al., 2006) or a gravity based foundation (Rahman et al., 1977). Under extreme conditions, the buildup of pore pressure will lead to liquefaction. Then, the areas of potential pore water pressure build up



on a caisson type of foundation will be: the area in front of the breakwater centered at the nodal sections and extending towards the anti-nodal sections (zone 1, Figure 1), and the area under the breakwater centered at the middle of the width and extending towards the onshore and offshore ends of the foundation (zone 2, Figure 1). Puzrin et al. (2010) investigated such a case where several caissons near Barcelona harbor had sunk into seabed due to storm waves.

The pore water pressure build-up due to the movement of a gravity based foundation can be produced by an overturning moment over a breakwater, which produces a kind of “rocking motion”. However, in order to reach liquefaction by the rocking motion large wave impacts have to be applied in order to generate significant overturning moment over the structure. Sumer et al. (2008) observed that pore water pressure build up due to rocking motion over a caisson was strongly dependent on the amplitude of the caisson motion. De Groot et al. (2006a) reported field observations of pore water pressure build up on the Ekofisk oil storage tank in the North Sea after a storm, although the pore pressure build up was not sufficient to cause liquefaction. Other case histories where liquefaction took place in marine structures have been reported by Zen et al. (1986), and Chaney & Fang (1991).

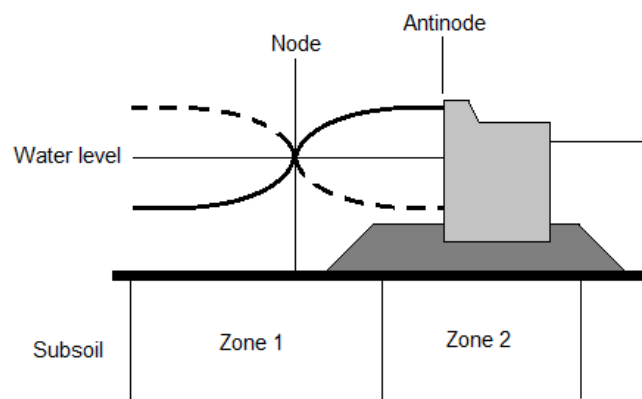


Figure 1. Caisson breakwater and potential liquefaction zones (Modified from Sumer, 2014)

A commonly used foundation for marine structures, and especially for offshore wind turbines are the monopiles, which are feasible to install for water depth up to 30 m. The high cyclic lateral moments applied to these structures, due to wind and wave loading, are transferred to the soil through lateral resistance of the embedment (Cuellar, 2011). Strong cyclic loads produced by a storm or an earthquake, may generate a buildup of pore water pressure which can affect the pile's capacity and stability. After this event, small cyclic lateral loads will allow the excess pore water pressures to dissipate and therefore the soil surrounding the monopile will densify.

The process of pore pressure buildup and subsequent liquefaction can also affect the stability of stone protections used in marine structures, such as scour protections (Sekiguchi et al., 2000; Sumer et al., 2010), and rock berms. The experiments of Sekiguchi et al. (2000) and Sumer et al. (2010) showed that cover stones such as the ones used for scour protections sink when the seabed liquefies. Yet, having a cover of stones above the liquefiable layer increases the liquefaction resistance as the

stones densify the soil. Rock berms are installed over a pipeline or a power transmission line/cable to protect them from floating. After installing the rock berm the trench may be backfilled with in situ sediment intentionally or by sediment transport. Due to this backfilling process this sediment will be in a loose state and therefore prone to liquefaction. If liquefaction takes place the rock berms will be subjected to the motion of the liquefied soil. As well, if rock berms are installed on a pipeline lying on a loose sediment, and therefore prone to liquefaction due to wave action, the berm structure will also be exposed to motions of the liquefied soil (Sumer, 2014).

Recently, Kirca (2013) studied the sinking of an individual cover/armor stone (i.e. irregular shaped heavy object) due to wave-induced liquefaction. Results indicated that the shape of the block is of minor (or no) importance from the dynamics of sinking standpoint. Based on their experimental data and the earlier studies, Kirca (2013) suggested a tentative procedure to assess the sinking damage (ultimate sinking depth) of an object on the seabed in the case of residual liquefaction. A broader discussion of the subject is given Sumer (2014) with assessment examples of sinking for different individual heavy objects.

The following sections will describe the procedures for the assessment of liquefaction potential, the mechanism of wave-induced liquefaction, and the effect of clay content on liquefaction susceptibility.

## 5.1 Liquefaction Hazard

Seabeds are susceptible to liquefaction and macroscopic degradation of their mechanical properties, which in turn is dangerous for offshore structures because of potential excessive settlements, sinking, or the loss of stability. Therefore, it is important for proper designing of marine infrastructure to recognize the liquefaction susceptibility of seabed in relation with interactions with structures and water.

Different approaches have been developed in order to evaluate liquefaction hazard of granular soils (Seed & Idriss, 1971; Ishihara, 1977; Dobry et al., 1982; Iwasaki et al., 1984). In the previous references the liquefaction potential is analyzed in terms of the soil properties. The soils that are more susceptible to liquefaction are saturated sands and silts. In terms of density, a loose deposit will experience strain softening due to a monotonic or cyclic load, when the shear stresses applied are larger than the ultimate shear strength of the soil. For this condition the loose soil experiences flow liquefaction, deforming at a low constant residual stress. For a dense deposit, the soil experiences strain hardening due to cyclic undrained loading. In this case cyclic mobility may take place as the dense soil generates negative excess pore water pressures.

The properties that make a granular soil more or less susceptible to liquefaction are the density, confining stress, soil structure (fabric, age, and cementation), the magnitude of the cyclic loading, and the extent of stress reversals.

Seed & Idriss (1971) developed a simplified procedure for the evaluation of the liquefaction resistance of soils due to earthquake loading. A state of the art report used as a standard for liquefaction hazard assessment was done by Whitman (1985, NRC). Later on, Youd & Idriss (1997) updated the simplified procedure.

The simplified procedure for liquefaction hazard assessment is based on empirical evaluations of field observations and laboratory tests. Two parameters are needed for the evaluation: the seismic demand on a soil layer (CSR: critical stress ratio), and the capacity of the soil to resist liquefaction (CRR: cyclic resistant ratio). The evaluation of the CSR is done by expression 1 (Seed & Idriss, 1971), where  $a_{max}$  is the peak horizontal acceleration at the ground surface,  $g$  the acceleration of gravity,  $\sigma_{vo}$  and  $\sigma'_{vo}$  are the total and effective vertical overburden stresses, respectively, and  $r_d$  is the stress reduction coefficient which account for the flexibility of the soil profile. The modifications done to the original expression of Seed & Idriss (1971) were by changing the values of  $r_d$ .

$$CSR = 0.65(a_{max} / g)(\sigma_{vo} / \sigma'_{vo}) \cdot r_d \quad (5.1)$$

For the evaluation of the liquefaction resistance a correlations are done based on field test results such as the cone penetration test (CPT), the standard penetration test (SPT), the Becker penetration test (BPT), and shear wave velocity measurements. To obtain the CRR in terms of field tests a series of charts have been developed to correlate the test characteristic parameters to the CRR (Youd & Idriss, 2001; Idriss & Boulanger, 2010).

The triggering of liquefaction is then analyzed by comparing the CSR and CRR, and obtaining a factor of safety  $FS = CRR/CSR$ . For the evaluation of liquefaction potential produced by wave loading a similar method than the previous one can be employed. Similar than the shear stresses applied by an earthquake wave loading will produce a gradual pore water pressure build up in the soil which may lead to liquefaction (Sumer, 2014). Peacock & Seed (1968) and Alba et al. (1976) performed a series of cyclic simple shear tests to study pore water pressure build up in sand. The previous authors obtained an empirical expression relating the shear stresses applied and the mean effective stress of the soil with the number of cycles  $N_l$  to reach liquefaction. The value of  $N_l$  will mainly depend on the amplitude of the oscillating shear stress acting on the soil  $\tau$ , the mean effective stress  $\sigma'_o$ , and density of the soil. Sumer (2014) developed a method to describe the buildup of pore water pressure due to wave loading based on the previous authors. Expression 2 calculates the number of loading cycles required to cause liquefaction  $N_l$ , where the ratio  $\tau/\sigma'_o$  is the normalized shear stress (shear stress / mean effective stress), and  $\alpha$  and  $\beta$  are empirical coefficients. In this case, the soil is considered completely undrained. Sumer et al. (2012) gives an empirical expression for the coefficients  $\alpha$  and  $\beta$  in terms of relative density. Recently, Jeng and Zhaou (2014) have modified this approach with a time dependent shear stress term. Their solution was seen to be close to that of Sumer et al. (2012).

By using Biot equations Sumer (2014) (earlier Sumer and Fredsoe, 2002) developed a mathematical analysis to obtain the stresses in the soil as well as the pore water pressures under a progressive wave (for a detailed analysis of these expression refer to Sumer, 2014). In this way, the generated excess pore water pressure can be compared with the mean effective stress of the soil to see whether under the wave loading and soil conditions liquefaction is possible.

Another way to estimate liquefaction potential due to wave loading is to correlate the relative density of the soil with SPT or CPT results. In this way the resistance of soil to liquefaction can be calculated similar to the CRR of the simplified procedure (Seed & Idriss, 1971; Youd & Idriss, 2001).

There is scarce literature regarding the liquefaction potential assessment of the soil interacting with a structure. Based on case histories of caisson foundations, de Groot et al. (2006b) developed a chart where four different types of liquefaction failures are induced by wave loading. The diagram in Figure 2 is in terms of the relative density  $Dr$ , and the ratio of the characteristic drainage time  $T_{CHAR,DRAIN}$  and the wave period  $T$ , where the characteristic drainage time is defined as expression (5.2).

$$T_{CHAR,DRAIN} = \frac{d^2}{c_v} \quad (5.2)$$

Where:

$d$ : Thickness of the subsoil layer, measured from the seabed to the impermeable base,  
 $c_v$ : coefficient of consolidation.

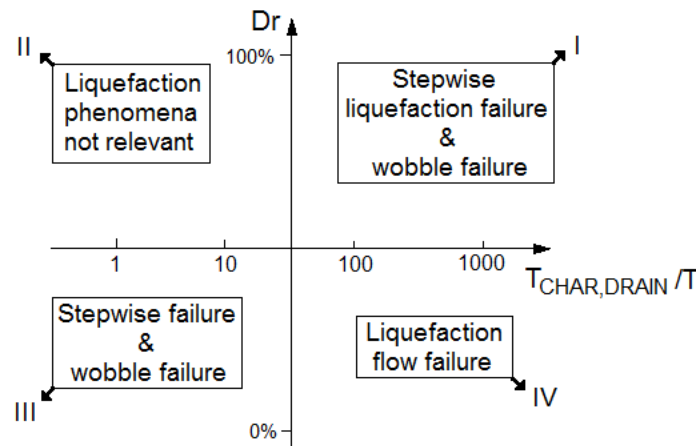


Figure 2 Potential failure types as a function of relative density and relative characteristic drainage period (replicated from de Groot et al., 2006b).

The ratio of the characteristic drainage time and the wave period can be understood as the time required for the water to drain out of the soil as a results of the wave loading. Therefore, as this ratio becomes larger, the soil is more susceptible to residual liquefaction.

The four types of failure mechanisms of a caisson foundation due to the action of wave loading are defined in the following (de Groot et al., 2006; Sumer, 2014):

*I Liquefaction flow failure:* Due to one or a few extreme wave loads large continuous deformations are generated in the foundation, as a results of a reduction in soil strength caused by accumulated pore pressures.

*II Stepwise liquefaction failure:* Significant residual accumulated pore pressures cause failure through stepwise residual caisson displacements.

*III Stepwise failure:* Failure through stepwise caisson displacements without significant residual accumulated pore pressures. This type of failure may be related to the failure mechanism produced by momentary liquefaction.

*IV Wobble failure:* Large cyclic shear strains produce failure by very large phase-resolved deformations.

The diagram developed by de Groot et al. (2006b), and shown in Figure 2, can be used as a preliminary assessment of liquefaction potential under a caisson or gravity based structure. At the same time, Sumer (2014) developed an approximate mathematical model which can be adopted to verify liquefaction potential. Refer to Sumer (2014) for detailed description of the model.

## 5.2 Wave-Induced Liquefaction

Marine soils are constantly exposed to wave action. The liquefaction phenomenon is essentially associated with large waves. These waves may have a period of 5–15 s, with 50 or 100 year-return-period wave heights as much as 1–2 m or larger in coastal areas, and 10–20 m in offshore areas. Marine soils that can be subjected to liquefaction are basically fine sands and silts, or a mixture such as silty sand or clayey sand (Sumer, 2014).

Liquefaction is generated mainly by two different mechanisms in waves (Sumer & Fredsoe, 2002; Sumer, 2014): a buildup of pore pressure (residual liquefaction), and an upward-directed vertical pressure gradient in the soil during the passage of a wave trough (momentary liquefaction). Several experimental studies have observed the phenomenon of wave-induced liquefaction, such as: Tzang (1998), Teh et al. (2003, 2006), and Sumer et al. (1999, 2006, 2012, 2014). The observations of wave flume tests performed by Sumer et al. (1999, 2012) have identified three primary factors that affect the accumulation of pore water pressures in the soil: the cyclic shear stress ratio (ratio of shear stress and mean effective stress), the period of cyclic loading, and the number of loading cycles required to reach liquefaction. Wave flume tests were also performed by Sawicki et al. (2014xx) to provide empirical data for testing different models of the seabed. A theoretical analysis of the models was done by considering classical soil mechanics, including the Biot type approach. Different theoretical descriptions have also been attempted by different authors such as: Been & Jefferies (1985), Ishihara (1985), Lade & Yamamuro (1997), or Sawicki & Mierczynski (2006).

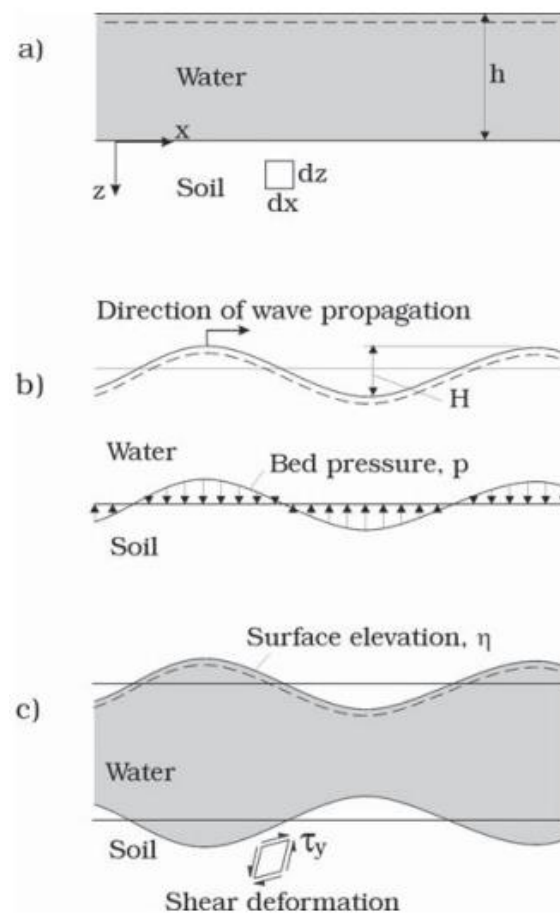
The following sections include a more general description of residual and momentary liquefaction produced by wave loading.

### 5.2.1 Residual Liquefaction

When a loose granular soil in undrained conditions is subjected to a progressive wave the pore pressure will gradually build-up, leading to liquefaction as the excess pore water pressure reaches a critical value. Residual liquefaction is a progressive process, occurring first at the surface of the seabed and then progressing downwards. After liquefaction the soil starts to compact, starting from the base of the liquefiable layer and progressing upwards. Figure 1 shows the mechanism of a progressive wave and its effect on the seabed soil. The seabed soil will be subjected to a variation of pressures due to the progressive wave (Figure 1b), as the pressures are different under the wave crest and the wave trough. An element of soil (Figure 1c) will then experience shear stresses,

varying periodically with time as the wave loads pass. If the seabed soil is loose and at undrained conditions it is possible that pore water pressures starts to build up.

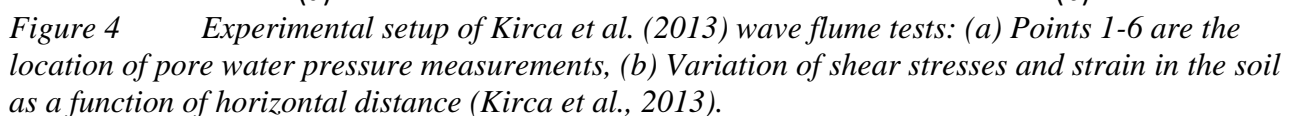
Experimental investigations studying the residual liquefaction process in a granular soil has been reported by Sekiguchi et al. (2000) and Miyamoto et al. (2004) on centrifuge tests, and Sumer et al. (2004, 2006), and Kirca et al. (2013) on wave flume tests. The study of wave-induced liquefaction or build-up of pore pressure in silt has been reported by Clukey et al. (1983, 1985), Foda & Tzang (1994), Tzang et al. (1992), Tzang (1998), and Sumer et al. (1999), all previous studies undertaken in a laboratory wave flume. Similar experimental studies for cohesive soils have been reported by Wit & Kranenburg (1992), and Kessel & Kranenburg (1998).



*Figure 3 Elastic deformation of the seabed soil under a progressive wave (Sumer, 2014): (a) Element of soil and initial conditions without wave loading, (b) Periodic pressure variation of the soil under a progressive wave, (c) Generation of shear stresses in the soil.*

Standing waves can also produce residual liquefaction, affecting breakwaters, seawalls, and large caisson structures. These waves are formed by the combination of the reflected waves meeting the incoming waves (Dean & Dalrymple, 1984). Standing waves and liquefaction has been studied by Sekiguchi et al. (1995), Sassa & Sekiguchi (1999, 2001), and Kirca et al. (2012, 2013). Figure 2

The process of pore water pressure build up at the node was described by Kirca et al. (2013) as a rearrangement of the soil grains due to cyclic shear strains, and at the same time the pore water is pressurized leading to a build-up. The accumulated pore water pressure results in liquefaction of the seabed and then spreads towards the antinode section. Similarities between the mechanism of standing waves and progressive waves were observed in the previous tests.



- Centrifuge tests performed by Sekiguchi et al. (2000) found that liquefaction progressed downwards when applying severe wave loading. As well, protecting the surface of the liquefiable layer with gravel suppressed liquefaction taking place. The later finding was confirmed by Sumer et al. (2010) on wave flume tests.
- The sequence of residual liquefaction was studied by Sumer et al. (2004, 2006) with tests performed on a standard wave flume. The excess pore pressure accumulated with the wave action until it reached a critical level, and the soil liquefied. The tests also showed how liquefaction was first developed at the mudline and progressed downwards. These results were in accordance with those from Sassa & Sekiguchi (1999), and Miyamoto et al. (2004). Sumer et al. (2006) also observed that the entire layer of soil was liquefied down to the impermeable base. Teh et al. (2003) and Sumer et al. (2011) observed that when the depth of the loose layer is

- Miyamoto et al. (2004) and Sumer et al. (2006) studied the process of densification (solidification, compaction) of the liquefied layer, the first authors using a small wave tank in a centrifuge, and the latest authors using a wave flume. The densification process progresses upwards during continued wave loading, and as a result of the densification the mudline will continuously move downwards (Sumer, 2014). The total time at which the excess pore pressure has completely dissipated will coincide with the time at which the compaction front arrives at the mudline (Sumer, 2014). At this condition, no further settlement is experienced by the soil.
- Sumer et al. (2006) observed that as the wave height increases so does the shaking of the liquefiable soil, and therefore the densification is larger. The relative density of the soil changed from a loose condition to dense after the liquefaction/compaction sequence of the tests.
- Sumer et al. (2006) observed that after the compaction stage, waves produced ripples in the soil surface (Figure 4) which reached a certain height and then stayed constant. These ripples were similar in steepness for a sediment without previous liquefaction.
- Sumer (2014) compared Sassa & Sekiguchi's (1999) wave centrifuge tests with those from Sumer et al. (2006) wave flume test results. By considering non-dimensional parameters the average pore pressure of the soil at a certain depth on the wave flume tests was compared by the centrifuge field" experiments, giving very similar results, and therefore proving that wave flume tests can be used as a physical model test for studying buildup of pore pressure and liquefaction analysis, provided that the results are analyzed and interpreted on the basis of non-dimensional parameters (Sumer, 2014).
- Liquefaction under Standing waves was studied by Sassa & Sekiguchi (1999) on centrifuge tests, and Kirca et al. (2013) on wave flume tests. Both references observed liquefaction phenomena occurring at the nodal section and spreading to the antinodal section. It was found that the number of waves to cause liquefaction at the nodal section was the same from that experienced by progressive waves for the same wave height (Kirca et al., 2013).
- Sekiguchi et al. (1995) derived a closed-form solution considering 1D poroelastoplasticity under Standing waves, and Sassa & Sekiguchi (2001) presented a 2D constitutive model in a finite element analysis, comparing their numerical results with their centrifuge tests (Sassa & Sekiguchi, 1999).

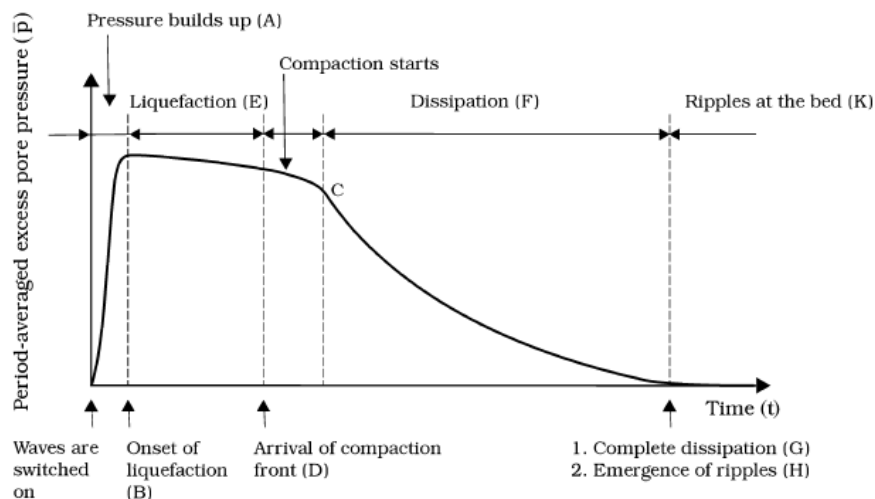




Figure 5. Time series of period-averaged excess pore water pressure at depth  $z$ . Schematic description (Sumer et al. 2006).

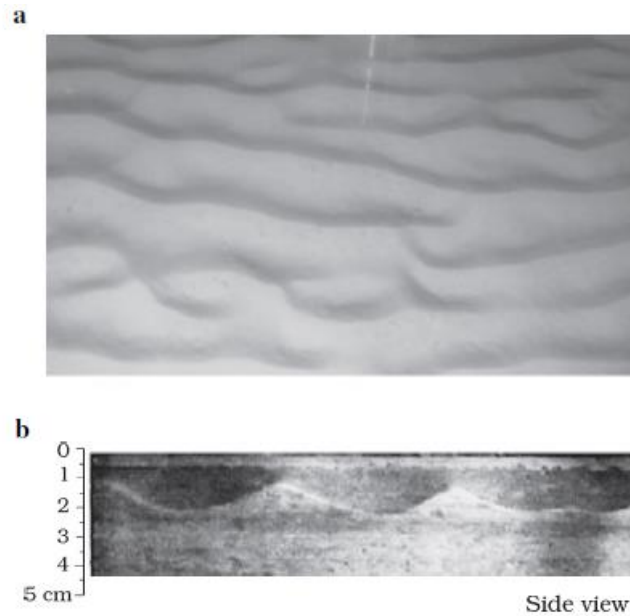


Figure 6. Bed ripples: (a) plan view, (b) side view. (Sumer et al. 2006).

### 5.2.2 Momentary Liquefaction

This type of liquefaction occurs over a short period of time during the passage of a wave trough. Under the wave trough the excess pore water pressure will be negative and therefore induce a lift force on the seabed (Figure 5). If the lifting force exceeds the submerged weight of the soil, this will fail the soil, and as a result it will liquefy.

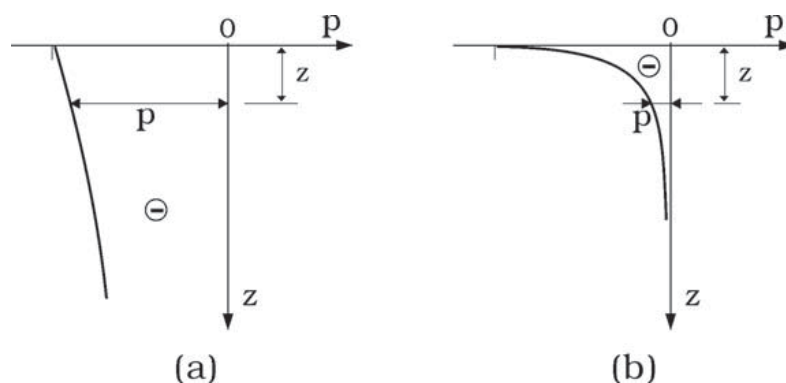


Figure 7. Pore water pressure distribution (in excess of hydrostatic pressure) during the passage of a wave trough. (a) Saturated soil. (b) Unsaturated soil (Sumer, 2014).

Sumer (2014) (earlier Sumer and Fredsoe, 2002) performed an analytical solution to evaluate momentary liquefaction on a saturated soil. It was found that momentary liquefaction will not occur

in a completely saturated soil. Similar conclusions were given by Hsu & Jeng (1994). For the case of an unsaturated soil containing air/gas, the vertical pressure gradient can be very large, particularly at small depths, and therefore the lifting generated at the soil surface can be significant (Sumer, 2014). Studies on unsaturated soil and momentary liquefaction have been undertaken by Sakai et al. (1992a) limited to shallow waters, and Sumer (2014) with an analytical analysis.

For the occurrence of momentary liquefaction it seems that an unsaturated soil is essential. Evidence of air and gas bubbles in marine soils has been registered by Hattori et al. (1992), Sakai et al. (1992), Mory et al. (2007), and Torum (2007). Zen & Yamazaki (1990a) performed an experimental study of momentary liquefaction and densification of the seabed. The results of the previous authors indicated that by pressurizing the water in a cyclic manner momentary liquefaction occurred at the seabed. As well, very large settlements and densification were measured resulting from the sequence of liquefaction and compaction.

### 5.3 Influence of Clay Content

The seabed may contain cohesive soils or a composite material, such as clayey silt or clayey sand (Kirca et al., 2014). Kirca et al. 2014 conducted wave flume tests in order to study the influence of clay content on wave-induced liquefaction. Two mixtures were considered: mainly silt-clay, and some complementary tests with sand-clay. The experiments by Kirca et al. (2014) showed that clay content has a significant influence on wave-induced liquefaction of sand and silt. It was observed that up to a clay content of roughly 30% the liquefaction susceptibility of silt was increased. For clay contents higher than 30% the silt-clay mixture was not liquefied. For the sand-clay mixture there was dependence on the size of sand grains: for a sand with  $D_{50} = 0.4$  mm. the mixture was liquefied by adding 10.8% of clay, whereas for a sand with  $D_{50} = 0.17$  mm. the mixture was partially liquefied by adding only 2.9% of clay. The authors linked the previous results with soil permeability and its influence on the buildup of pore water pressure. For the case of the silt-clay mixture with a clay content less than 30%, Kirca et al. (2014) explains that the soil had a low plasticity behaviour as referred by Gratchev et al. (2006), allowing the soil grains to rearrange under the cyclic shear strains, and therefore leading to a build-up of water pore pressure. By increasing further the clay content ( $> 30\%$ ) the permeability of the mixture decreases significantly and the soil grains are not able to rearrange, and the mixture behaves as a high plasticity soil, with a higher resistance to liquefaction. The results obtained by Kirca et al. (2014) show that they are dependent on the type of clay used, and therefore the clay content limit of 30% for the silt-clay mixture should not be taken as a constant threshold value (Sumer, 2014).

The practical applications of the previous study is that by following standard liquefaction potential analysis methods, the soil's permeability has to be assessed as liquefaction susceptibility is directly linked to this parameter (Kirca et al., 2014).

Sawicki et al. (2014) performed an experimental investigation to study the effects of fines in sand on liquefaction susceptibility, albeit not under wave loading. Their findings suggest that fines do not essentially influence the liquefaction susceptibility of sand.

Other studies have concentrated in purely clay sediments, with laboratory results suggesting that waves can liquefy these sediments, producing high density mud layers (Linderberg et al., 1989; de Wit & Kranenburg, 1992; de Wit, 1995).

## 6 Stability of Scour Protection Layers during Liquefaction Events

The phenomenon of scour around offshore foundations may occur due to the effect of currents, the combined effect of waves and currents, and due to seismic action. To reduce the effect of scour around offshore foundations a certain type of protection is needed. The most common type of scour protection used for offshore foundations consist of an armour layer of stones. Many site investigations indicate that numerous structural failures can be attributed to the effects of wave-induced liquefaction of the seabed, which will affect the scour protection. Previous studies on the effect of liquefaction over scour protections of stones has been focused on the wave-induced liquefaction (Sumer et al. 2010; Sekiguchi et al. 2000).

Sekiguchi et al. (2000) studied a fine silt covered partially and completely with gravel. The tests were performed on a small wave tank installed on a centrifuge. During the tests a steady seepage flow was maintained. When the sand was fully covered by a layer of gravel it did not liquefy. Sekiguchi et al's tests demonstrated that a cover of stones could prevent liquefaction induced by waves on a loose bed of fine sand.

Sumer et al. (2010) did similar experiments on a wave flume, using silt and a cover of stones. Figure 6 shows the time series of pore water pressure at a depth of 16 cm for four different tests. In the first case (Figure 6a), the liquefiable soil has no cover of stones and the pore pressure gradually builds up until liquefaction occurs. A similar result was obtained when using one and two layers of cover stones (case b and c in Figure 6). These results were also confirmed for depths at 5, 10 and 13 cm. In the last case (Figure 6d) where three layers of cover stones were used, liquefaction did not occur. The previous results were concluded to be a cause of densification of the soil with the cover of stones, and as the cover layer thickness increased so did the density of the soil. Figure 7 plots the results of Figure 6, in terms of the number of waves to reach liquefaction (liquefaction resistance) against the relative density of the soil. It is observed that even though cases (b) and (c) in Figure also liquefied, the number of waves to reach liquefaction was increased compared to cover layers, case (a) Figure 6.

No references were found regarding the effect of seismic-induced liquefaction on scour protections. Currently, the University of Dundee centrifuge laboratory, as part of the MERMAID project are undertaking an experimental study of liquefaction produced by earthquakes and its effect on cover stones. The tests consist of a layer of fine loose sand, covered by a layers of stones. The number of layers or surcharge applied to the soil will be modified. The model is tested at 50g level and fully saturated. As well, a similar test program is underway at ITU on a shaking table equipment for comparison purposes. Again fine loose sand (see Table 1) in a fully saturated state is used for these tests. For start, the exerted shaking is on a single horizontal axis, sinusoidal with a peak acceleration of 0.5g. In the following stages, additional degrees of freedom will be added to the shaking (horizontal+horizontal or horizontal+vertical). The test results will then be compared with the ones performed by Sekiguchi et al. (2000) and Sumer et al. (2010).

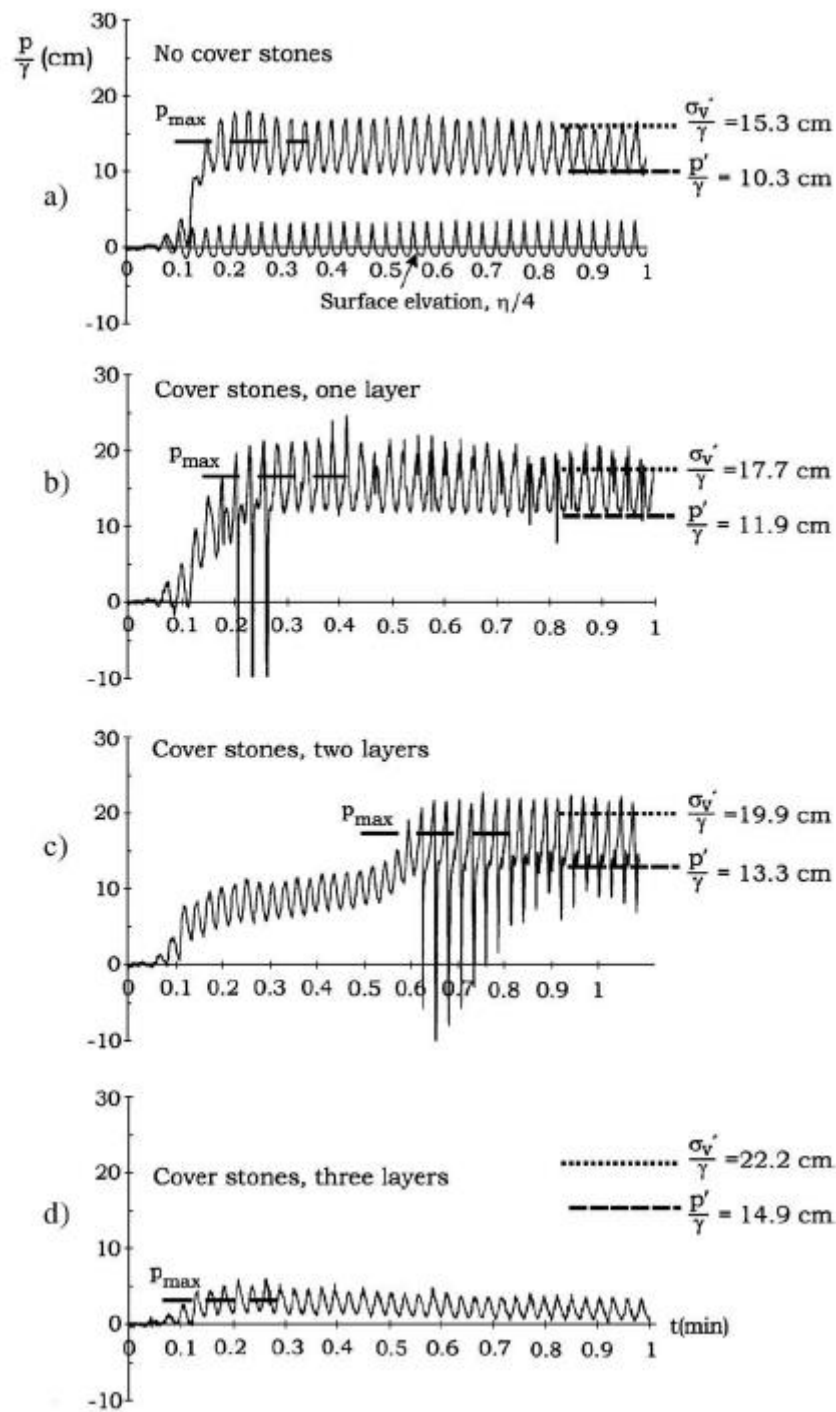


Figure 8. Time series of excess pore pressure. Influence of cover stones. Sumer et al. (2010).

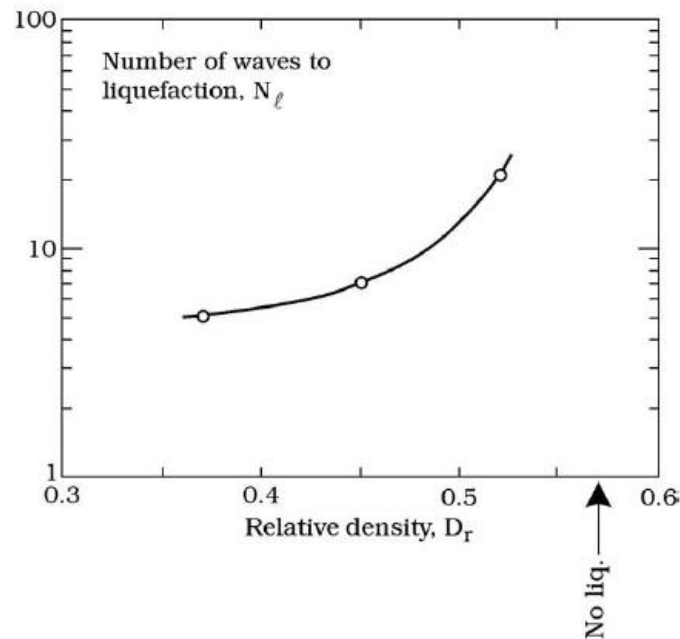


Figure 9. Liquefaction resistance as a function of the soils relative density. The first data point on the left: no cover stones. Middle data point: one layer of stones. Third data point: two cover layers. Arrow sign of “No Liq.”: three cover layers (Sumer et al., 2010).

## 7 Summary of Guidelines

Based on the work carried out under the auspices of MERMAID, the following guidance is provided:

- Sandy soils have a high liquefaction potential, the effect of which must be accounted for in foundation design.
- The presence of small quantities of fines within a sandy soil can act to *increase* the potential for wave-induced liquefaction. For the soils studied, this occurred for fine fractions up to 30% of total mass. Sandy soils with larger quantities of fines, or sandy clays, were less susceptible as the clay behaviour dominates above this fraction.
- Liquefaction risk under gravity base foundations should be screened properly in the light of in-situ and lab tests of the seabed soil. The preliminary screening methods in the literature (some of which cited here) can be used as a first assessment. If these assessments indicate any risks, detailed analyses and models should be applied for liquefaction assessment.
- When it comes to wave-induced liquefaction, standing waves would be seen in the vicinity of reflecting boundaries of platform foundations. Liquefaction under Standing waves, although qualitatively similar, show features different from that caused by progressive waves. The pore pressure accumulation and liquefaction starts at the nodal section, and progresses towards anti-nodal section due to a diffusion mechanism. The rate of liquefaction at the nodal section seems to be the same with the progressive wave case. Thus, mathematical models for progressive waves would also work for standing waves as far as the nodal section is concerned.
- Rock berms (protection cover) on pipelines and power transmission cables are essential to protect the structure from floating to the seabed surface due to liquefaction, and strongly suggested to be constructed. However, once the rock berm is constructed, care must be

exercised on backfilling the excavated soil into the trench intentionally or due to sediment transport. Such a loose backfill will be prone to liquefaction, and if liquefaction takes place, the motion of the liquefied soil may presumably damage the rock berm more severely than the wave motion on the bare berm itself.

- Studies showed that a cover of stones constructed around the platform foundations for scour protection purpose would also serve for increasing the liquefaction resistance of surrounding soil. Once the stone cover is designed with respect to hydraulic loads and filtering requirements, an additional check is suggested to be performed to assess the effect of the stone cover on increasing the resistance to wave-induced liquefaction, so that a fine-tuning can be performed on the dimensions or layer thickness of the stone cover if deemed necessary.
- The influence of earthquake shaking and seismic-induced liquefaction on scour protection of platforms has not yet been particularly studied in the available literature. But, there are significant studies underway in the scope of MERMAID project the results of which are expected to shed light on this problem, once published.

A state-of-the-art of “Liquefaction around Marine Structures” can be found in the book with the same title, which covers many of the aforementioned subjects researched under MERMAID project and additional matters regarding liquefaction issues of platform designs (Sumer, 2014). The book itself serves as a compendium of guidelines and will be useful for designers and practitioners from this aspect.

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